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A HYDRAULIC WATER QUALITY MODEL OF SUISUN AND SAN PABLO BAYS

Water Resources Engineers, Incorporated
Lafayette, California

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A HYDRAULIC-
WATER QUALITY MODEL
OF
SUISUN AND SAN PABLO BAYS

REPORT OF AN INVESTIGATION
CONDUCTED FOR THE
FEDERAL WATER POLLUTION CONTROL ADMINISTRATION
DEPARTMENT OF INTERIOR

BY

WATER RESOURCES ENGINEERS, INC.
LAFAYETTE, CALIFORNIA

MARCH 1966

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LIST OF SYMBOLS

SYMBOL	DESCRIPTION	DIMENSIONS
α	Wave celerity	LT^{-1}
a	Exponent	-
b	Exponent	-
c	Concentration	-
c_s	Saturation concentration	-
C	Constant	-
C_3	Constant	-
d	Water depth	L
g	Acceleration of gravity	LT^{-2}
h	Head	L
K	Frictional constant	LT^{-2}
K_1	BOD reaction velocity constant	T^{-1}
K_2	Reaeration coefficient	T^{-1}
K_d	Eddy diffusion coefficient	$L^2 T^{-1}$
L	Biochemical oxygen demand, BOD	-
n	Manning's roughness coefficient	-
q	Discharge per foot of width	$L^2 T^{-1}$
r	Hydraulic radius	L
S_e	Energy gradient	-
t	Time	T
u	Velocity in x-direction	LT^{-1}
v	Velocity in y-direction	LT^{-1}
x	Coordinate axis	L
y	Coordinate axis	L

I INTRODUCTION

BACKGROUND FOR INVESTIGATION

San Pablo and Suisun Bays, as shown in Figure 1, are the connecting links between the Sacramento-San Joaquin Delta and San Francisco Bay. Suisun Bay is essentially an extension of the Delta with two shallow water areas lying to the north of the main channel. It has a surface area of about 30,000 acres at mean tide (MT) and a mean tidal prism of 115,000 acre-feet. Suisun and San Pablo Bays are connected by the seven-mile long Carquinez Straits which has a mean tidal prism of about 20,000 acre feet.

San Pablo Bay is a much larger embayment with a water surface area of approximately 75,000 acres at MT and a mean tidal prism of 320,000 acre-feet. The main channel through the Bay lies along its south-eastern edge with the vast shallow portion of the bay lying to the north. The general shallowness is evidenced by the fact that 57 percent of the bay has a depth of less than six feet below mean lower low water (MLLW).

Both of these shallow bays receive the sediment-laden waters passing from California's Central Valley through the Delta and serve as depositories for particles which are flocculated under the influence of the lower estuary's salinity. Continuous dredging operations are required to maintain depths in the main channel adequate for navigational purposes.

Considerable interest in both the quantity and quality of these waters has been shown in recent years by private individuals and organizations and

FIGURE 1

SAN FRANCISCO, SAN PABLO, AND SUISUN BAYS

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local, state and federal agencies. This is evidenced, in part, by the State Water Quality Control Board's comprehensive study of San Francisco Bay, the formation in 1965 of the Bay Conservation and Development Commission (BCDC), the Water Pollution Control law of 1965 enacted by the Legislature of the State of California, and the Central Pacific River Basin Comprehensive Study being conducted by the Federal Water Pollution Control Administration (FWPCA).

The State Water Quality Control Board (SWQCB)* contracted with the Sanitary Engineering Research Laboratory of the University of California to conduct a comprehensive study of San Francisco Bay. This study, which was carried out over 7 years from 1958 through 1965, represents one of the most massive data collection and interpretation programs ever undertaken to evaluate the water quality condition of an estuarine water body. The area studied comprised all of San Francisco, San Pablo and Suisun Bays and their connecting channels as far upstream as Antioch at the lower extremity of the Sacramento-San Joaquin Delta.

Formation of the Bay Conservation and Development Commission (BCDC) was initiated with the passage of Senate Bill 14 in the 1964 special session of the Legislature of the State of California. Specifically, the Commission was given the responsibility to

(a) ascertain the public interest in San Francisco Bay;

(b) study the effects of further filling of the Bay; and

(c) recommend to the Legislature measures to protect the public interest in the bay.

The Water Pollution Control Law of 1965, passed by the California State Legislature during July of that year, directed the SWQCB to develop a preliminary report and prospectus for a study which could produce a master plan for water quality control in San Francisco Bay, the Sacramento-San Joaquin

*Prior to 1963, the State Water Pollution Control Board (SWPCB)

Delta areas and their tributary drainages. The preliminary report, submitted to the State Legislature in February, 1966, detailed the scope and objectives of the study and indicated a budget requirement of about $2.8 million for a two and one-half year program. This figure was predicated on developing the cooperation and collaboration of other state, local, and federal organizations engaged in similar or specialized studies of the Bay-Delta system.

Recently, the FWPCA initiated its comprehensive study of water quality problems of the Central Pacific Basin, including the area drained by the Bay-Delta system. An important activity of this program has been the development and application of a series of mathematical models for predicting the response of particular elements of the system to the various water resource development and water quality management plans being considered and/or implemented by State and Federal water agencies. The development of these models has been a collaborative undertaking of the FWPCA[1] and Water Resources Engineers, Inc. (WRE) of Lafayette, California, implemented partly by FWPCA-supported research contracts.

The models, in compatible form for both hydrodynamic and water quality studies, were initially developed for and applied to the complex Sacramento-San Joaquin Delta system of channels[2]. Under a recent contract[3], the basic model structures were developed by WRE for Suisun and San Pablo Bays, verified for a selected set of hydrologic conditions for the entire system from the upper limits of the Delta through San Pablo Bay, and checked for functional

1 Formerly the Division of Water Supply and Pollution Control, Public Health Service, DHEW

2 "A Water Quality Model of the Sacramento-San Joaquin Delta.", Report of an Invest. for U. S. Public Health Service, Water Resources Engineers, Inc., Lafayette, California, June 1965.

3 No. PH 86-65-114 USPHS

water quality behavior. The present report describes activities carried out under the terms of the contract and presents preliminary results of applications of the models.

SCOPE AND OBJECTIVES

On June 18, 1965 Water Resources Engineers, Inc. entered a contract with the Division of Water Supply and Pollution Control, U. S. Public Health Service, presently the Federal Water Pollution Control Administration, to provide professional services for the development and evaluation of a mathematical hydraulic-water quality model of Suisum and San Pablo Bays. In part, the contract states

A. . . . . "Sepcifically, it (the model) shall be constructed to give the following information:

   (1) The histories of tidal elevation at selected boundary and internal points of the system, including locations of known gaging stations and,

   (2) The histories of tidal velocity at internal points in sufficient detail to permit definition of current structure and circulation in the system.

B. Investigation leading to development of the hydraulic model and its use in study of water quality problems shall consist of the following phases:

   (1) design or selection of an appropriate two-dimensional prototype including definition of physical and hydraulic parameters.

   (2) representation of the prototype in model form as a system of "one-dimensional" channels.

   (3) preparation of a test problem for solution using the existing Delta hydraulic program.

(4) testing to determine conformance with realistic prototype behavior including adjustment of boundary conditions.

(5) evaluation of the effect of boundary configuration and development of methods of treating physical prototype boundaries in the model.

(6) verification of adjusted model on prototype test case.

(7) testing and verification of the water quality model developed for the Delta under Contract No. PH 86-64-145, using data produced by the hydraulic model.

ORGANIZATION

The development of the hydraulic-water quality model of Suisun and San Pablo Bays was accomplished by WRE personnel working in close collaboration with the Federal Water Pollution Control Administration. Dr. G. T. Orlob served as principal investigator, Mr. D. E. Evenson was responsible for the development of the model, and Mr. M. R. Lindorf coordinated all engineering activities for the project. Mr. James C. McCarty served as Project Director for the FWPCA and the basic data for the water quality simulations were provided by his staff. Overall project direction was the responsibility of Mr. Paul W. Eastman, Regional Program Director, FWPCA. Computer services were provided to the project by arrangement with CEIR , Inc. of San Francisco, California.

ACKNOWLEDGEMENTS

Water Resources Engineers, Inc., wishes to acknowledge its appreciation for the splendid cooperation and support it received from personnel of the FWPCA Region IX Office, in particular, that of Mr. Paul W. Eastman and Mr. James C. McCarty, whose encouragement and confidence contributed to the success of the project.

II HYDRAULICS

THE PROBLEM

Suisun and San Pablo Bays are large, shallow embayments which together provide storage capacity within the mean tidal prism for about 450,000 acre-feet of water. Both direction and magnitude of tidal and fresh water flows within these embayments are variable with time as frictional and gravitational forces act upon the masses of fluid in circulation.

During periods of flood tide these shallow bays accumulate in storage the large volumes of saline and fresh waters entering through the limiting boundaries of the system. As the tide ebbs, relatively fresh water continues to enter the system through the Delta and a mixture of salt and fresh water is released from storage to be distributed to downstream locations from which they will return during the succeeding flood tide. Conditions which govern the hydrodynamic behavior of the system are essentially those of the tide at the lower boundary, the inflow at the upper boundaries, and the extractions which may occur at intermediate boundary locations.

When a pollutant enters the system through a river, a drain, an outfall, or from the ocean its distribution will be determined by the hydraulic mechanisms of advection, dispersion, and eddy "diffusion". Each of these mechanisms is operative along all principal axes, although to significantly different degrees along some axes than others. For example, the shallow character of a bay suggests that vertical mixing may be very rapid and that pollutant distribution may be largely determined by movement in the horizontal plane.

Hence, it may be considered sufficient for some practical purposes to consider the problem as "two-dimensional". It is important to recognize in this connection that the assumption of mathematical two-dimensionality does not remove consideration of the water depth; it merely considers that vertical velocity components and turbulent fluctuations are relatively unimportant.

Field observation of currents and salinity distributions confirm that San Pablo and Suisun Bays are well-mixed vertically. While there are some indications of stratification in the deeper portions of the lower embayment, especially at times of high surface runoff, it is not expected that treatment of the current dynamics as two-dimensional would result in any serious departure from prototype behavior.

ANALYTICAL APPROACH

Mathematical characterization of the hydrodynamics of a two-dimensional system requires the simultaneous solution of both the dynamic equation of flow and the continuity equation. The theoretical bases for these equations are well known and adequately described in the literature[4], hence, it should suffice for this report to state them in the form in which they will be treated.

The dyanmic equations of motion in two-dimensions are:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial h}{\partial x} + K|\mathbf{v}|u = 0 \qquad (1)$$

$$\frac{\partial v}{\partial t} + v\frac{\partial v}{\partial y} + u\frac{\partial v}{\partial x} + g\frac{\partial h}{\partial y} + K|\mathbf{v}|v = 0 \qquad (2)$$

4 Dronkers, J. J., Tidal Computations in Rivers and Coastal Waters, North Holland Publishing Co., Amsterdam, 1964.

where

V	=	$(u^2 + v^2)^{1/2}$
x, y	=	coordinate axes
u	=	velocity in the direction of the x-axis
v	=	velocity in the direction of the y-axis
h	=	water surface elevation
g	=	acceleration of gravity
K	=	frictional term
t	=	time

The equation of continuity is expressed as

$$\frac{\partial h}{\partial t} + \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = 0 \qquad (3)$$

where

q	=	discharge per unit width which is equal to the product of a depth, d, and a mean velocity corresponding to the appropriate coordinate axis.

Each of the terms in the equations of motion can be physically identified. Local acceleration (the rate of change of velocity) is identified by the first term of the equation while the directional accelerations are contained in the second and third terms. Gravitational force or potential is expressed by the fourth term and the frictional force opposing motion is described in the fifth.

The continuity equation represents the rate of change of water surface elevation induced by changes in the rates of flow in the x and y directions.

In order to completely define the hydrodynamic behavior and the distribution of pollutants within the combined Sacramento-San Joaquin Delta - San

Pablo - Suisun Bay System*, the model developed for the embayment portion of the system must be compatible in structural characteristics with that previously developed for the Delta. There exist at least two alternatives whereby compatibility between models can be established. They are:

(1) Two separate models could be employed in which the tidal output of a Suisun - San Pablo Bay model would be supplied as the tidal input to the Delta model. Conversly, the discharge derived from the Delta model would become input for the Bay model. Hence, a "trial and error" solution would be required to assure satisfaction of common boundary conditions.

(2) A single comprehensive model of the system could be employed incorporating both the Delta and Bay systems.

The scope and objectives of the WRE-FWPCA contract were developed initially on the basis that the first alternative would be the most feasible, since it was anticipated that computer core storage requirements for a single model would exceed the capacity of available hardware. Subsequently, however, it was considered that the advantages of a single model of the system might be sufficient to justify some further examination of this alternative. A review of the problems presented indicated that with certain structural modifications and economies in programming the complete system could be incorporated in one model. This alternative yields some economic advantage in that some of the "trials" required by the first alternative are avoided. Offsetting this advantage, however, is the increase in cost associated with developing and verifying the larger model of the complete system.

In view of the ultimate goals of the project in which the models are to be used, it was decided that the second alternative was, in fact, the most attractive. As a result of this decision, however, it was necessary also to review the

* The so-called "Bay-Delta System"

original Delta model to determine the feasibility of a general two-dimensional approach as opposed to representing the complete system as a mesh of one-dimensional elements.

Restructuring the Delta Model in a two-dimensional form would require the formidable undertaking of assigning vector properties and coordinates for all elements of the system. Rather than embark on this uncertain course it was decided first to check the applicability of the one-dimensional equations of flow, as applied to the Delta system, to a two-dimensional embayment. The applicable one-dimensional equations which were used in designing the original hydrodynamic model of the Delta System are:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + g\frac{\partial h}{\partial x} + K|u|u = 0 \qquad (4)$$

and

$$\frac{\partial h}{\partial t} + \frac{\partial q}{\partial x} = 0 \qquad (5)$$

A comparison of Equations (3) and (5) reveals that continuity can be satisfied by either a two-dimensional approach or by a one-dimensional representation of the two-dimensional problem. However, in comparing Equations (1) and (2) with (4) it is evident that important differences exist in the directional acceleration and frictional terms.

The relative importance of the several terms can be illustrated by calculation using representative values of velocities and gradients experienced in the real physical system. The range in expected absolute value and the relative size of each of the terms in Equation (4), as shown in Table 1, confirms that the directional acceleration terms are relatively unimportant and the potential and frictional forces are dominant. The influence of the frictional term in

Term / Magnitude	$u\frac{\Delta u}{\Delta x}$	$g\frac{\Delta h}{\Delta x}$	$K\|u\|u$
absolute, x 10^6	0-20	$\pm$ 300	$\pm$ 1000
relative	0.02	0.30	1.00

TABLE 1 ABSOLUTE AND RELATIVE MAGNITUDE OF THE TERMS IN THE DYNAMIC FLOW EQUATION

the one and two-dimensional approaches is identified with the absolute value of the velocity. Since,

$$(u^2 + v^2)^{1/2} > u$$

it follows that, other factors being equivalent the one-dimensional equations will underestimate the frictional resistance; the amount depending upon the direction of the velocity.

Considering this effect, the absence of reliable resistance coefficients, and the complications of dealing with the two-dimensional form of the equations, it was decided that the one-dimensional equations of flow could be practically applied to the system. The merit of this approach would, of course, have to be proven through actual simulation of the dynamic behavior of the two-dimensional system being modeled.

III NUMERICAL METHODS

INTRODUCTION

Exact solutions to the partial differential equations that describe the dynamics of unsteady flow, Equations (4) and (5), are not known except for the simplest of cases. Consequently, in order to obtain a solution the equations must be written in discrete form and numerical methods of integration employed.

Numerical methods of integration are of two general classes, involving either explicit or implicit procedures. In an explicit scheme each equation, in discrete form, is written so that all variables, except one, are described by their value prior to integration. Thus, each of the unknowns in a sequence of such equations can be solved for directly. However, in implicit methods each variable is expressed as the average of its values before and after the integration interval. As a result, each unknown is coupled with other unknowns and a set of simultaneous equations results. The number of equations to be solved simultaneously is equal to the number of unknowns.

Inherent in the numerical integration of partial differential equations is the problem of maintaining a stable solution. The stability requirements of explicit methods are rigid and impose a limit on the maximum size of the integration interval. In implicit schemes the unknowns are coupled and a significant relaxation of the stability requirement is achieved. As a result much larger integration intervals can be employed.

In general, when numerically integrating over a specified interval an

explicit scheme will result in comparatively simple equations which must be solved repeatedly; whereas an implicit procedure will produce more complicated equations requiring less frequent solution.

An explicit method of numerical integration was used in the Sacramento-San Joaquin Delta Hydrodynamic Model. Physical limitations imposed by prototype configuration plus the computational stability requirement limited the interval of integration in this model to 2.5 minutes. The expansion of the Delta model to include the embayments, thereby nearly doubling the size of the system, points up the need to increase the integration interval and to reduce accordingly the cost of future production runs. For this reason an investigation of implicit methods of integration was considered worth while. Such a study offered the attractive possibilities of relaxing stability requirements and decreasing computer time per solution.

In order to fully illustrate the differences between these two numerical procedures for equation solving, the concepts involved in each will be applied to the dynamic equations of flow. The advantages of each will be discussed in the following sections of this report with special reference being made to their practical application to the Bay-Delta System.

EXPLICIT METHOD

Details of the explicit scheme used in the Sacramento-San Joaquin Delta Model have been presented previously[5] and will not be reintroduced here. However, some basic concepts involved in developing this scheme need to be presented for illustrative and comparative purposes.

5 Shubinski, R. P., J. C. McCarty, and M. R. Lindorf, "Computer Simulation of Estuarial Networks", Proc. ASCE, Jour Hyd. Div., Sept. 1965.

Expressing Equations (4) and (5) in terms of discrete increments results in

$$\frac{\Delta u}{\Delta t} + u\frac{\Delta u}{\Delta x} + g\frac{\Delta h}{\Delta x} + K|u|u = 0 \qquad (6)$$

$$\frac{\Delta h}{\Delta t} + \frac{\Delta q}{\Delta x} = 0 \qquad (7)$$

where

$$\frac{\Delta u}{\Delta t} = \frac{u^* - u}{\Delta t}$$

$$\frac{\Delta h}{\Delta t} = \frac{h^* - h}{\Delta t}$$

and the asterisk (*) indicates the properties of the system after the time interval Δt. The unknowns are u^* and h^*, which can be solved for explicity as

$$u^* = u - \left(u\frac{\Delta u}{\Delta x} + g\frac{\Delta h}{\Delta x} + K|u|u\right)\Delta t \qquad (8)$$

and

$$h^* = h - \frac{\Delta q}{\Delta x}\Delta t \qquad (9)$$

Thus, a point by point solution can be carried forward in time for each of the unknowns. A system containing n elements, each with the unknowns u^* and h^* would then require solving both Equations (8) and (9) n times.

As was demonstrated in the development of the Delta Tidal Hydrodynamic Model, the stability criterion for the numerical solution of Equations (8) and (9) is expressed as

$$(\alpha + u)\frac{\Delta t}{\Delta x} \leq 1.0 \qquad (10)$$

where α is the wave celerity. For the Delta Model this constraint, in addition

to the physical limitations placed upon the system as a result of its size, limited the time interval of integration, Δt, to 2.5 minutes.

It should be noted that in the explicit method employed with this model a modified Runge-Kutta procedure was used, incorporating half-steps to take into account the nonlinearity of Equations 4 and 5.

IMPLICIT METHOD

Implicit methods of numerical integration of partial differential equations have been successfully applied to unsteady gas flow through porous media[6] and to heat flow problems[7]. Basically, in the implicit formulation each term in the partial differential equation to be solved is written as the average of its value before and after the time interval Δt. Applying this principle to Equations (4) and (5) there results

$$\frac{(u^* - u)}{\Delta t} + \frac{(u^* + u)}{2}\,\frac{(\Delta u^* + \Delta u)}{2\Delta x} + g\frac{(\Delta h^* + \Delta h)}{2\Delta x}$$

$$+ \frac{K}{2}\,(|u^*|u^* + |u|u) = 0 \qquad (11)$$

6 Bruce, G. H., D. W. Peaceman, H. H. Rachford, Jr., and J. D. Rice, Calculations of Unsteady-State Gas Flow Through Porous Media, Petroleum Trans. AIME, Vol. 198, 1953, pg 79.

7 Peaceman, D. W. and H. H. Rachford, Jr., The Numerical Solution of Parabolic and Elliptic Differential Equations, J. Soc. Indust. Applied Math., Vol. 3, No. 1, March, 1955.

$$\frac{(h^* - h)}{\Delta t} + \left(\frac{\Delta q^* + \Delta q}{2\Delta x}\right) = \frac{(h^* - h)}{\Delta t} + \frac{\Delta(u^*d^*) + \Delta(ud)}{2\Delta x} = 0 \qquad (12)$$

where $q = ud$

d = water depth

Examining Equations (11) and (12) it can be seen that there are several non-linear terms in the unknowns u^* and h^*. To avoid computational complexities resulting from these nonlinear terms, each was replaced with linear combinations and Equations (11) and (12) were rewritten as

$$\frac{(u^* - u)}{\Delta t} + \frac{1}{2}\left(u^* \frac{\Delta u}{\Delta x} + u \frac{\Delta u^*}{\Delta x}\right) + g \frac{(\Delta h^* + \Delta h)}{2\,\Delta x} + K|u|u^* = 0 \qquad (13)$$

and

$$\frac{(h^* - h)}{\Delta t} + \frac{\Delta(u^*d) + \Delta(ud^*)}{2\Delta x} = 0 \qquad (14)$$

Thus, the non-linearities were eliminated and Equations (11) and (12) were reduced to a set of linear equations.

Approximations to the x-derivatives can be obtained by Taylor expansions about the i^{th} point. Neglecting third order and higher terms results in

$$\frac{\partial y}{\partial x} \simeq \frac{y_{i+1} - y_{i-1}}{2\Delta x} = \frac{\Delta y}{\Delta x} \qquad (15)$$

Substituting this approximation into Equations (13) and (14) and expanding, these equations can be written as

$$\left[-\frac{u_i \Delta t}{2\Delta x}\right] u^*_{i-1} + \left[1 + \frac{1}{2}\frac{\Delta u_i}{\Delta x}\Delta t + K|u_i|\Delta t\right] u^*_i + \left[\frac{u_i}{2}\frac{\Delta t}{\Delta x}\right] u^*_{i+1}$$

$$-\left[\frac{g}{2}\frac{\Delta t}{\Delta x}\right] h^*_{i-1} + \left[\frac{g}{2}\frac{\Delta t}{\Delta x}\right] h^*_{i+1} = u_i - \frac{g}{2}\frac{\Delta t}{\Delta x}\Delta h \qquad (16)$$

and

$$\left[-\frac{u_i}{2}\frac{\Delta t}{\Delta x}\right] h^*_{i-1} + h^*_i + \left[\frac{u_i}{2}\frac{\Delta t}{\Delta x}\right] h^*_{i+1}$$

$$-\left[\frac{d_i}{2}\frac{\Delta t}{\Delta x}\right] u^*_{i-1} + \left[\frac{d_i}{2}\frac{\Delta t}{\Delta x}\right] u^*_{i+1} = h_i \qquad (17)$$

Considering again a system with n elements, each with the unknowns u* and h*, and writing these equations for each of the elements results in a set of 2n linear simultaneous equations.

When systems become large, such as that of the Delta which involves 1196 unknowns, iterative procedures become the only feasible means of solving the resulting set of simultaneous equations. Preliminary results of studies of implicit methods indicate that significant savings in computer time, as a result of large increases in the time increment Δt, can be realized for one-dimensional systems and small two-dimensional systems. However, for large two-dimensional systems, such as the composite of the Delta and the embayments modeled under this contract, the efficiency in solving the resulting set of equations is reduced.

EXPLICIT VS IMPLICIT

Prior to making a decision regarding the choice of a numerical procedure to be used, the advantages and disadvantages of each must be evaluated

in light of the problem to be solved. Applying an explicit scheme to the dynamic equations of flow results in fairly simple equations which must be solved frequently. The frequency with which they must be solved is controlled by stability requirements with little regard to the size and complexity of the physical system. On the other hand, an implicit method results in a set of simultaneous equations which must be solved at less frequent intervals. However, the efficiency associated with solution to these equations is primarily dependent upon the size and complexity of the system. Stability requirements are relatively unimportant.

Realizing the size and complexity of the problem to be solved, the exploratory nature of the investigation of implicit procedures, and in view of the previous experience with the Delta System, it was decided to proceed with an explicit formulation. Further development and application of the implicit technique, which will be very valuable for certain classes of problems, will be reserved for the future.

IV HYDRAULIC AND WATER QUALITY MODELS

GENERAL

In recent years, with the advent of modern high-speed digital computers, it has been possible to develop mathematical models of extremely large and complex physical systems. In addition to enhancing the size and scope of problems which can be solved through their use, certain other advantages result from applications of mathematical models. Among these are low cost of development, short time and relatively low cost for simulation, ease of adapting models to changes in prototype systems, and flexibility in output.

Two basic mathematical models for solving hydrodynamic and water quality problems in an estuarine environment were previously developed for application to the Sacramento-San Joaquin Delta[2]. These models were constructed in a general form so that they could be applied with only slight modification to any comparable physical system, for example, that with which this report is concerned - Suisun and San Pablo Bays.

Adapting and applying the models to the Suisun and San Pablo Bay system is essentially a three-phase process. First, the physical system must be properly described. Second, the hydraulic properties of the system must be defined, the hydrodynamic model revised and the system simulated for specified hydraulic boundary conditions. And third, the dynamic water quality model must be modified and the system simulated under a given set of water quality boundary conditions.

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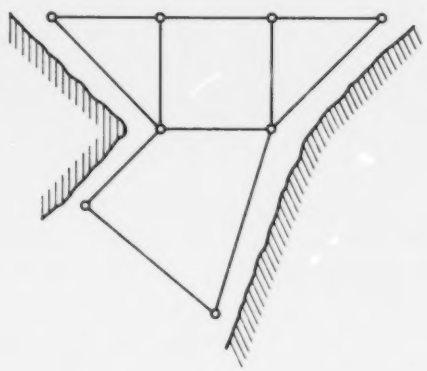
PHYSICAL SYSTEM

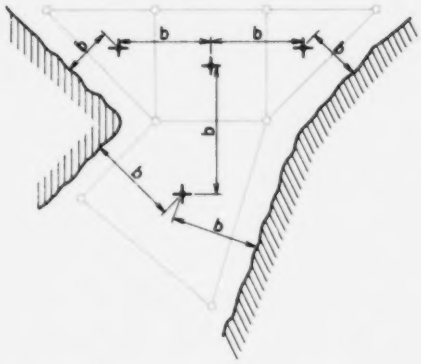
It is implicit in the mathematical modeling technique that the system be described as a network of finite elements whose behavior is defined by integrating numerically the partial differential hydrodynamic and water quality equations. For the Bay-Delta System the one-dimensional equations of flow and continuity are applied to a set of interconnected "channels" and "junctions" which are selected to represent the physical environment. Each of these channels and junctions is characterized by certain mathematical parameters of that portion of the prototype which they represent. Figure 2a shows a typical two-dimensional space as it may be represented by a system of junctions and one-dimensional connecting channels. Channel and junction properties as they were used in developing the models are described as follows:

Physical properties which are required to adequately define a typical channel are its characteristic length, width and depth. The length of a channel is simply the distance between the junctions which the channel connects. The width of a channel, as shown in Figure 2b, is defined as the distance between centroids of adjacent areas, which are bounded by lines connecting adjoining junctions, measured perpendicular to the axis of the channel. Widths for those channels represented in Figure 2a are shown in Figure 2b. Channel depth is taken as the mean depth of the rectangular area represented by the channel length times its width.

Each junction is defined physically by its surface area, depth, and volume. The surface area of a junction is that area bounded by the lines which connect the centroids of adjacent areas. Surface areas of the junctions, shown in the examples of Figure 2a, are indicated in Figure 2c. The volume of each junction is defined as the surface area times a depth which is computed as a weighted average of the mean depths for each of the channels entering the junction. Weighting factors are in proportion to the relative areas contributed by the several channels to the total surface area identified with the junction.



(a)



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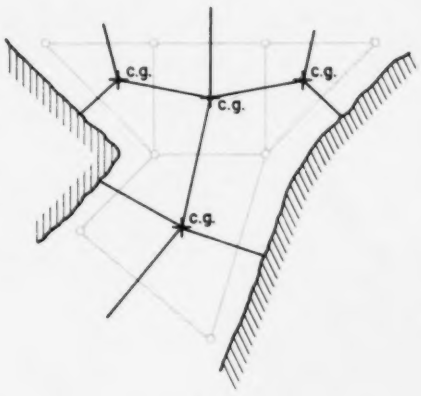

(c)

FIGURE 2

TYPICAL CHANNEL AND JUNCTION ELEMENTS

Networks of interconnected channels and junctions developed to represent Suisun and San Pablo Bays are shown in Figures 3 and 4, respectively. Channels were generally oriented in directions which would produce the smallest variation in depth between junctions. Suisun Bay was represented accordingly by a series of 125 junctions connected by 181 channels. The San Pablo Bay portion of the system being somewhat larger, required 131 junctions and 235 channels. Modeling the entire system including the Delta results in a total of 828 junctions and 1041 channels.

TIDAL HYDRODYNAMIC MODEL

Prior to application of the tidal hydrodynamic model certain hydraulic properties of the system must be defined. Those properties which require definition are the hydraulic radius, r, and the frictional resistance, K.

The hydraulic radius of a channel was assumed in the present instance to be equivalent to its mean depth at mean tide. This assumption may be justified by the fact that all channels are relatively shallow and broad and by recognizing that friction is absent along the sides of all channels except those adjacent to principal prototype boundaries. For the channels in question width to depth ratios were generally greater than 100.

Frictional resistance is represented in Equation (4) by the term

$$K|u|u = g S_e$$

where g = acceleration of gravity and
S_e = energy gradient.

Assuming that hydraulic conditions remain steady over the integration time interval the energy gradient may be evaluated by using Manning's Equation,

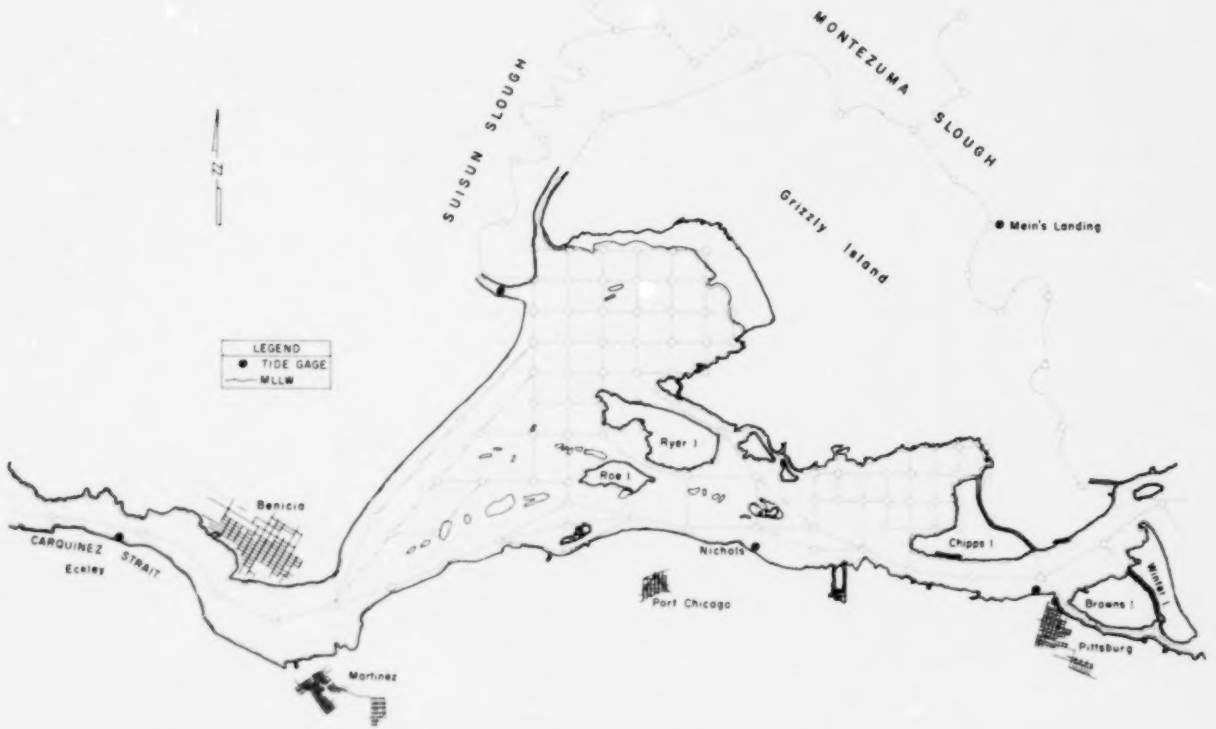

FIGURE 3

SUISUN BAY NETWORK

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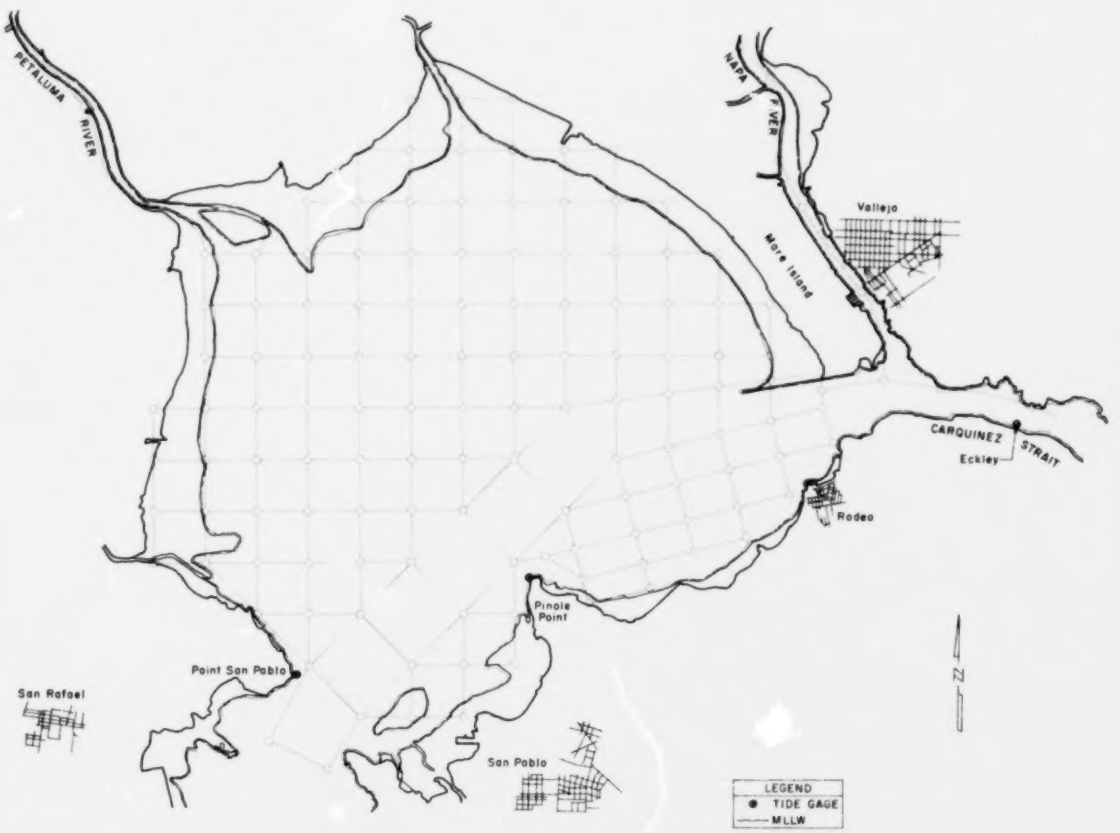

FIGURE 4

SAN PABLO BAY NETWORK

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32-B

$$S_e = \frac{n^2}{2.21\, r^{4/3}} |u|u$$

where n is Manning's roughness coefficient and r is the hydraulic radius. The constant, K, in the frictional resistance term can then be described as

$$K = \frac{g(n)^2}{2.21\, r^{4/3}}$$

Values of Manning's roughness coefficient which were used for modeling of Suisun and San Pablo Bays ranged from 0.018 to 0.025. These were somewhat less than were used in the Delta Model a condition which is consistant with the increased size of channels used and with the smoother boundaries encountered in this portion of the prototype.

The entire modeled system may be separated logically into three distinct physical environments; the Sacramento-San Joaquin Delta, Suisun Bay, and San Pablo Bay. Models for each of these environments require independent verification against prototype behavior. Verification of the model for the Sacramento-San Joaquin Delta has previously been accomplished[2] and is not treated in this report. Verification of models for Suisun and San Pablo Bays under stipulated boundary conditions is, of course, required in order to put the entire model in operative condition for simulation. The process of verification is discussed briefly in the remainder of this section.

Suisun Bay, which is just downstream from the Delta, was verified first. The necessary boundary conditions required to simulate the hydrodynamic behavior of Suisun Bay are those of the tide at the downstream boundary, namely, Carquinez Straits at Eckley, and the inflow/outflow between Suisun Bay and the Delta. Tidal conditions at the downstream boundary were obtained from a continuous record of the tidal elevation at Eckley. Inflow/outflow for Suisun Bay was estimated from current measurements made in the channel opposite Chipps Island near the upstream boundary. Using these boundary conditions, the system was simulated until a "dynamic steady state" condition existed, that is,

tidal fluctuations and current circulations were replicated in successive cycles.

Results from simulation of the prototype by the hydraulic model were compared to continuous records of tidal histories at selected gages within the system to insure correct model behavior. Both the continuous records and computed values from the model for four tide gages within Suisun Bay are shown on Figure 5; the locations of which are shown on Figure 3. These comparisons indicate that the model reproduced the measured tidal histories quite closely in both phase and amplitude. The largest discrepancies occurred during short periods near low tide when differences between model and prototype in the order of 6 to 9 inches were noted. Deviations during higher water periods were very small, generally less than 2 inches. Maximum deviations, which occurred over short intervals relative to the tidal period, were only about 1 to 2 percent of the water depth and hence could have only a minor influence on the discharge cross section. No significant discrepancies occurred in the phasing of the tide as it progressed through the model.

Considering the inherent inaccuracies in prototype measurements, the unsteadiness of flows, and the complexity of the system the resulting checks on tidal behavior were taken as satisfactory verification of the Suisun Bay portion of the model.

The San Pablo Bay Model was verified using as boundary conditions the measured tidal history at Point San Pedro at the lower end of the Bay and the inflow/outflow to the Bay from Carquinez Straits as generated in the verification run of Suisun Bay. Comparison of continuously recorded tidal histories at Pinole Point in San Pablo Bay and Carquinez Straits at Eckley, as shown in Figure 6, indicate excellent agreement between model and prototype (See Figure 4 for gage locations). Both the phasing and amplitude at Pinole Point agreed almost exactly with those measured. At Eckley there was a slight attenuation of the amplitude and the peaks appeared to occur slightly earlier than in the prototype. The difference between measured and calculated amplitudes was in

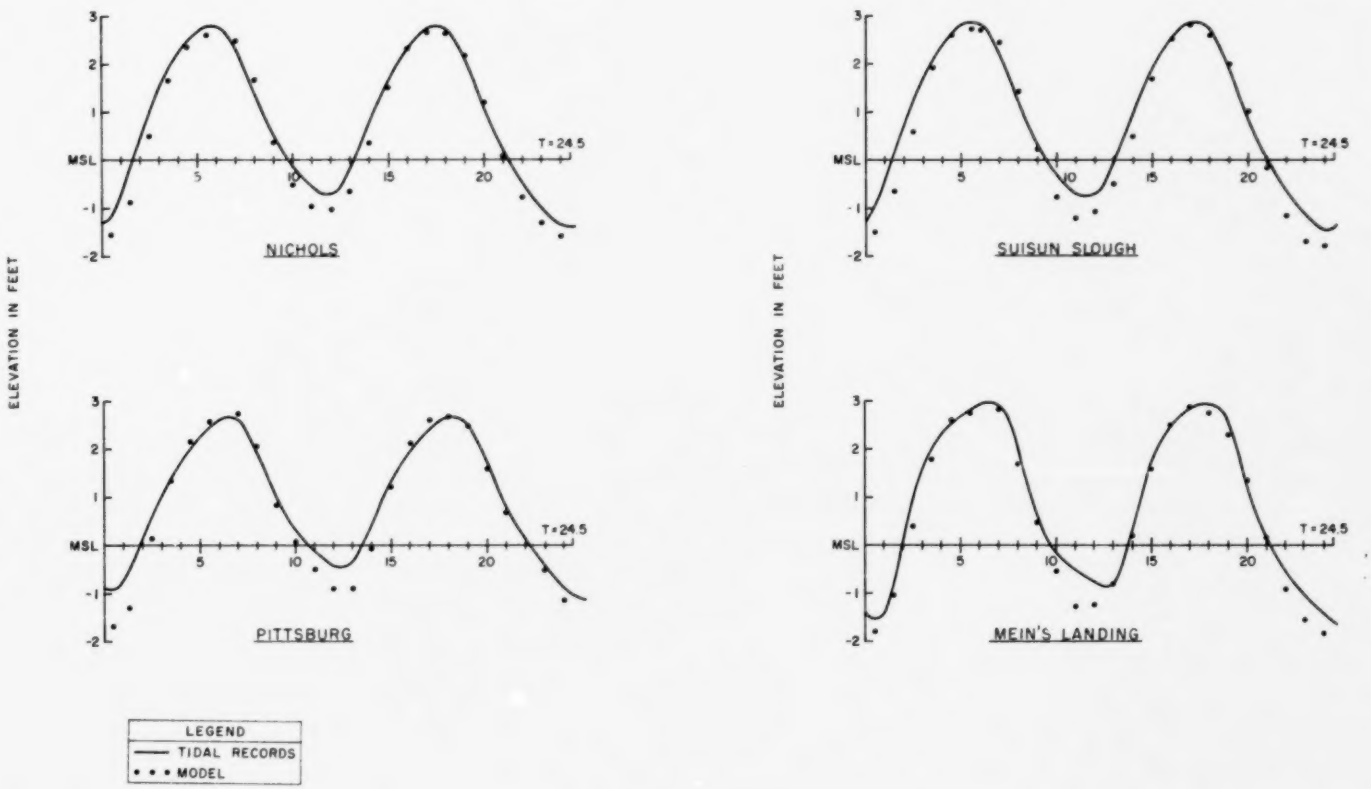

FIGURE 5

MODEL VERIFICATION OF TIDAL HISTORIES IN SUISUN BAY

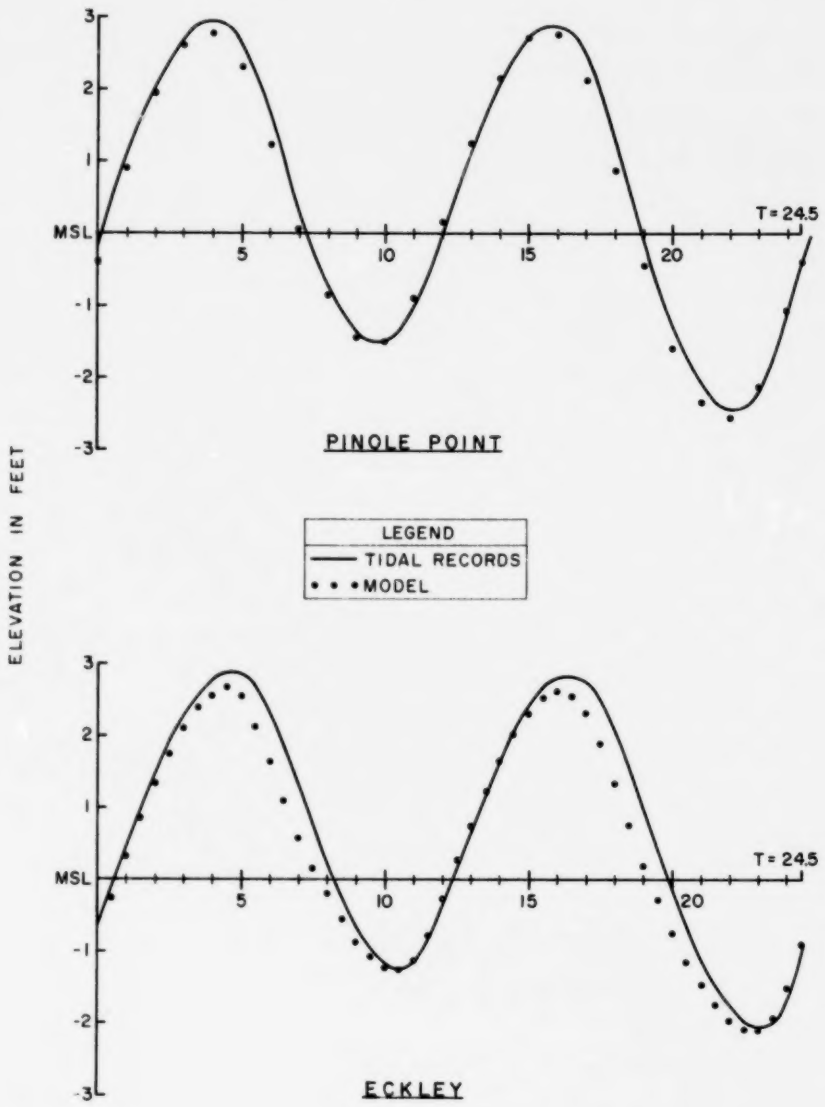

FIGURE 6

MODEL VERIFICATION OF TIDAL
HISTORIES IN SAN PABLO BAY

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the order of 3 to 4 inches, a deviation of relatively minor consequence in relation to its effect on discharge prediction. The calculated tidal elevations were for a point located approximately one mile downstream from the Eckley gaging station, hence a slight shift in phase between observed and computed values was expected.

Based upon these results the hydrodynamic behavior of both Suisun and San Pablo Bays was considered to be adequately defined. The adaptation and application of the water quality model to the entire system proceeded from this point.

DYNAMIC WATER QUALITY MODEL

The pollutional effects on the complex estuarial system comprised of the Delta, Suisun Bay, and San Pablo Bay are many and varied. However, in general these effects may be reduced to two categories; those dealing with "conservative" pollutants, such as may result from agricultural drainage or the intrusion of seawater into the lower Delta and those treating "non-conservative" pollutants such as the organic material carried into the Delta and Bays through tributary streams or discharged from domestic or industrial sources. Conservative pollutants introduced to the system are controlled in their spacial and temporal distribution by essentially two processes, advection and diffusion. Both of these processes are governed by the hydrodynamic properties of the system. Mathematically, the distribution of pollutant can be described by

$$\frac{\partial c}{\partial t} = K_d \frac{\partial^2 c}{\partial x^2} + u \frac{\partial c}{\partial x} \qquad (18)$$

where the first term is the local time rate of change of concentration and the second and third terms define the changes in concentration resulting from diffutional and advective processes, respectively.

Diffusional processes which affect the concentration of a pollutant are governed by the rate of change of the concentration gradient, $\partial^2 c/\partial x^2$, and the eddy diffusion coefficient, K_d. The eddy diffusion coefficient has been shown to be dependent upon the rate of energy dissipation and the scale of the phenomenon[8]. Expanding this concept, as illustrated previously[2], results in the following practical form for characterizing the eddy diffusion coefficient

$$K_d = C_3|u|d$$

The absolute value signs indicate that the direction of the transport by eddy diffusion is independent of the direction of flow and depends only upon the direction of the concentration gradient.

Concentration changes resulting from advective processes are dependent upon the concentration gradient and the velocity of flow. The importance of a compatible model which is capable of describing the hydrodynamics of the system is clearly evident as both the diffusional and advective processes are shown to be dependent upon velocity.

Non-conservative pollutants are characterized by changes in concentration resulting not only from diffusional and advective processes, but also from other physical processes which may deplete or supplement the pollutant as functions of time. In general, the distribution of a non-conservative pollutant is described as

$$\frac{\partial c}{\partial t} = K_d \frac{\partial^2 c}{\partial x^2} + u \frac{\partial c}{\partial x} + f(t) \qquad (19)$$

where f(t) indicates the functional relationship(s) which describe(s) the depletion and/or supplementation of the pollutant.

8 Orlob, G. T., "Eddy Diffusion in Homogeneous Turbulence" Trans. ASCE, Vol. 126, 1961, pp 397-438

An excellent example of a non-conservative substance is oxygen which is both depleted and supplemented. Biochemical stabilization of the organic material present in waters consumes oxygen during the oxidation process while reaeration supplements the oxygen resource in proportion to the oxygen deficit. The temporal and spacial distribution of oxygen is thus described as

$$\frac{\partial c}{\partial t} = K_d \frac{\partial^2 c}{\partial x^2} + u \frac{\partial c}{\partial x} + K_2(c_s - c) - K_1 L \qquad (20)$$

where $K_2(c_s - c)$ is the rate at which the oxygen is supplemented through reaeration and $K_1 L$ is the rate at which it is consumed by oxidation.

The process of reaeration through which oxygen is supplied to the system, is governed by the existing concentration, c, the saturation concentration, c_s, and the reaeration coefficient, K_2. Recently, there have been some significant investigations aimed at developing expressions for K_2 which recognize the importance of fluid turbulence in the reaeration process. Among these the most notable and practical are the studies of O'Connor and Dobbins[9], Churchill, et al[10], and Krenkel and Orlob[11]. Results of each of these investigations can be expressed in the same general form which relates the reaeration coefficient to the velocity, u, and water depth, d. This relationship is expressed as

$$K_2 = C\, u^a\, d^b$$

9 O'Connor, D. J. and Dobbins, W. E., "Mechanism of Reaeration in Natural Streams", Trans ASCE, v 123, 1958.

10 Churchill, M. A., Elmore, H. L., and Buckingham, R. A., "The Prediction of Stream Reaeration Rates", J. San Engrg. Div. ASCE v 88 July 1962

11 Krenkel, P. A. and G. T. Orlob "Turbulent Diffusion and the Reaeration Coefficient", J. San. Engr. Div. ASCE v 88 March 1962

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where C is a constant and a and b are exponents. A summary of the recommended values of these parameters is given in Table 2 for K_2 expressed as day^{-1}, u in feet per second and d in feet.

	C	a	b
O'Connor and Dobbins	12.9	1/2	-3/2
Churchill et al	11.5	1	-5/3
Krenkel and Orlob	2.5	1	- 1

TABLE 2 SUMMARY OF COEFFICIENTS FOR REAERATION EQUATION

Depletion of the oxygen resource through the presence of biodegradeable material is controlled by the amount of material present, L, and a reaction velocity constant, K. Since the biodegradeable material is a non-conservative pollutant its fate in the system is described generally by Equation (19). Its distribution in time and space is expressed in specific terms as

$$\frac{\partial L}{\partial t} = K_d \frac{\partial^2 L}{\partial x^2} + u \frac{\partial L}{\partial x} - K_1 L \qquad (21)$$

which must be solved simultaneously with Equation (20) in order to completely define the oxygen resource. The reaction velocity constant is generally in the range of 20 to 30 percent per day for domestic wastes discharged to natural receiving waters.

The partial differential equation(s) which describes the dynamics of water quality , Equation (19), must be solved conjunctively with the partial differential equations which describe the hydrodynamics of the system, Equations (4) and (5). As previously discussed in Chapter III, the only feasible means of obtaining a solution to these equations is by writing them in discrete form and employing numerical methods for their integration.

The basic structure of the water quality model was developed under a previous contract (No. PH 86-64-145) and described in the resulting report. However, at this point it is appropriate at least to summarize the several basic phenomena which are considered by the model and to state in discrete form for reference purposes the appropriate equations. They are as follows:

a. Advection

$$(\Delta c)_a = cu \cdot \frac{\Delta t}{\Delta x}$$

b. Diffusion

$$(\Delta c)_d = K_d \frac{\Delta c}{\Delta x} \frac{\Delta t}{\Delta x}$$

c. Reaeration

$$(\Delta c)_o = K_2(c_s - c)\Delta t$$

d. Degradation

$$(\Delta c)_b = K_1 L \Delta t$$

In addition to these processes the model accommodates pollutants added to the system at any of its junctions, water removed from the junctions at its existing concentration, and evaporated at zero concentration.

Under the present contract the hydrodynamic model of San Pablo and Suisun Bays was to be made compatible with the existing general water quality model, that is, the structure of the two models were to be identical. However, as discussed in Chapter II, it was decided to modify the goals of the present contract, and to combine both San Pablo and Suisun Bays with the Delta in one large system. It logically follows then that the entire system should be verified rather than just certain portions. This would require modification of the existing water quality model in order to incorporate the two-dimensionality of the Bays and

would, accordingly, add to the expense of verification. Considering long-range goals of the project and the purposes for which the model might ultimately be used, it was decided that this form of verification should be conducted. The larger scale model this produced was expected to be more versatile and more readily applicable to quality problems under study by the FWPCA.

The problem of sea water intrusion into the Bays and Delta was selected by the FWPCA for verification purposes. Since some field data was available for both the hydrodynamic behavior and salinity intrusion into these areas for the month of July, 1955, this period was considered most appropriate for simulation.

Prior to simulation the initial conditions of the system must be known. Based on the available data, limited as it was, a set of initial salinity concentrations were generated. These are shown on Figure 7a. For a given set of boundary conditions the entire system - Suisun Bay, San Pablo Bay, and the Delta - was then simulated both hydrodynamically and with respect to water quality for a period corresponding to 18 tidal cycles.

Results from this simulation, as shown on Figure 7a, give the distribution of salinity in the system after eighteen cycles of intrusion. During the period simulated the seawater intruded at a rate of approximately 0.4 miles per tidal cycle; the rate varying with the concentration.

However, just as important as the instantaneous distribution of salt is the maximum concentrations which occur during a particular tidal cycle. Figure 7b shows the maximum concentrations occurring during the eighteenth tidal cycle as predicted by the model as well as maximum concentrations observed in the field which occur at slack water after high-high tide.

Also important in evaluating the behavior of the model is the actual distribution of the mass of salt in the system. Concentrations were converted

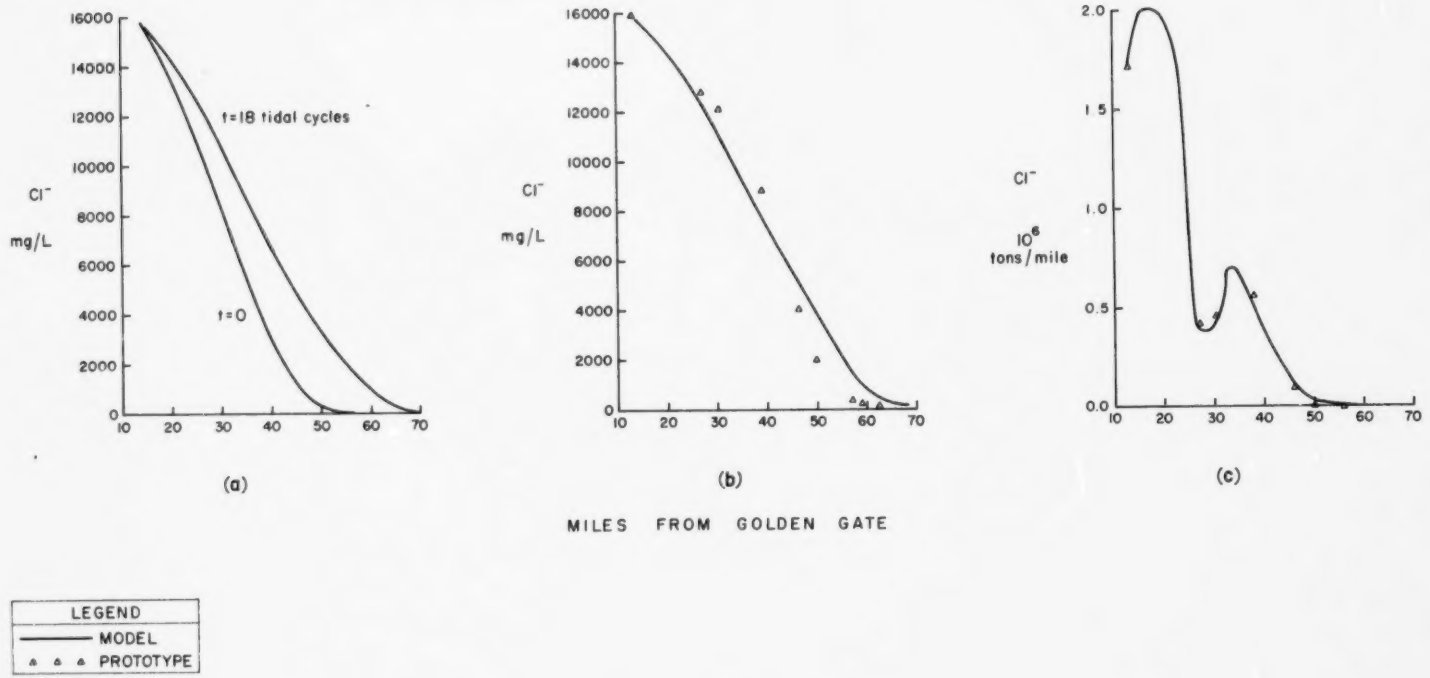

FIGURE 7

MODEL VERIFICATION OF SALINITY INTRUSION IN SAN PABLO BAY,

SUISUN BAY AND THE SACRAMENTO - SAN JOAQUIN DELTA

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to tons/mile and the results shown on Figure 7c.

Figure 7b indicates that the model predicts somewhat higher concentrations in the upper reaches of the system than were measured in the prototype under assumedly comparable conditions. On the other hand, considering the actual mass distribution of salt in the system the model agrees very well with the prototype. This apparent anomoly is explained by the fact that there is a relatively small volume of water in the upper reaches and thus, the mass of salt in these reaches is small relative to that contained in San Pablo and Suisun Bays.

Basically, there are four possible causes of the discrepancies between the values predicted by the model and those measured in the prototype. These possibilities are

1. Inadequacy of prototype data
2. Incorrect initial conditions for simulation
3. Incorrect boundary conditions
4. Induced advective dispersion.

The first three possible causes are largely independent of the model structure and indicate the need for more reliable field observations to support model development and verification. Both initial and boundary conditions were developed using what limited data was available.

"Induced advective dispersion" is a consequence of the use of numerical methods in integrating Equation (18). Considering only the advective term in Equation (18) the rate of change of concentration becomes

$$\frac{\partial c}{\partial t} = u \frac{\partial c}{\partial x}$$

which when written in discrete form is

$$\frac{\Delta c}{\Delta t} = \frac{c^* - c}{\Delta t} = u\frac{\Delta c}{\Delta x} \qquad (22)$$

Solving for the unknown concentration, c^*, at time $t + \Delta t$ Equation (22) becomes

$$c^* = c + u\frac{\Delta c}{\Delta x}\,\Delta t \qquad (23)$$

Again approximating the x-derivatives by Taylor Expansion and neglecting third-order and higher terms results in

$$\frac{\Delta c}{\Delta x} = \frac{c_{i-1} - c_{i+1}}{2\Delta x}$$

and substituting into Equation (23) yields

$$c_i^* = c_i + c_{i-1}\left(\frac{u\Delta t}{2\Delta x}\right) - c_{i+1}\left(\frac{u\Delta t}{2\Delta x}\right)$$

which describes numerically the advective process. Realizing that $u\Delta t$ is the distance a particle of water will move in the time increment Δt and that Δx is the actual distance moved in the numerical process it is evident that a pollutant can be transfered a distance either greater or less than the actual mean travel distance $u\Delta t$. This phenomena has been termed "induced advective dispersion" and is inherent in the numerical methods used in the water quality model. The exact magnitude of the error involved is difficult to assess due to the complexity of equations being solved and the size and scope of the system simulated. A sensitivity analysis of the models should prove extremely valuable in evaluating the effects each of these sources contribute to the final result.

An appraisal of the results presented in Figure 7 leads to the general conclusion that the two models, hydrodynamic and water quality, when applied to the combined Bay-Delta System are capable of simulating the quality changes which occur under dynamic conditions. Verification, even though satisfactorily achieved by the models, is limited by the adequacy of basic data rather than the functional capabilities of the mathematical model or the simulation techniques.

V SUMMARY AND CONCLUSIONS

The investigation described herein was directed toward the development of practical engineering tools for the study of water quality problems in San Pablo and Suisun Bays. During periods of low river flow sea water intrudes through these bays into the Sacramento-San Joaquin Delta. In future years, as fresh water is diverted through the Delta region to water deficient areas of Central and Southern California, it is expected that the amount and distribution of fresh water releases to this complex system will have a significant effect on the quality changes in the lower embayments.

The Federal Water Pollution Control Administration, because of its responsibilities in the field of water supply and pollution control, sponsored the development of a hydrodynamic model of Suisun and San Pablo Bays which was to be structurally compatible with a water quality model previously developed. The water quality model was designed originally to depict the temporal and spacial distribution of conservative and non-conservative pollutants in the Delta but was made generally applicable to estuarial channel systems.

An investigation of the equations which describe the hydrodynamics of Suisun and San Pablo Bays indicated that this two-dimensional system could be approximated by a system of one-dimension channels similar to that employed for the Delta. This approximation combined with structural modifications of the existing Delta Model and economies in programming permitted the entire system - Suisun Bay, San Pablo Bay, and the Sacramento-San Joaquin Delta - to be included in one large model.

As the size of the system increased the need for a more efficient computational scheme became apparent. The existing hydrodynamic program used an explicit procedure for numerically integrating the partial differential equations which define the hydrodynamics of the system. Stability requirements are rigid and severly limit the maximum size of the permissible integration interval. Implicit schemes of numerical integration, on the other hand, permit relaxation of stability requirements but result in more complicated equations to be solved. A preliminary investigation of explicit and implicit schemes indicated that implicit procedures could be used advantageously in one-dimensional and in small two-dimensional systems.

The size and two-dimensionality of the Bay-Delta system prevented the application of implicit methods. It was recognized, however, that the full potential of implicit procedures has not been fully realized and that additional efforts in this area could prove fruitful for future applications.

In development of the models both Suisun and San Pablo Bays were structured as networks of interconnected channels and junctions. These systems when combined with the previously developed Delta System resulted in a single model comprised of 1041 channels and 828 junctions. Tidal elevations, velocities, and discharges for all elements of the model were determined for specified hydrologic conditions by solving the one-dimensional equations of motions and continuity.

Verification of the Delta portion of the system was in a prior investigation. The Suisun and San Pablo Bay components were verified in the present study by comparison of model generated tidal elevations with observed elevations on the prototype. Current patterns in San Pablo Bay were confirmed qualitatively, although adequate field data for conclusive verification was lacking.

The validity of the Dynamic Water Quality Model for application to the combined Bay - Delta System was confirmed by simulating salinity intrusion through and into the Bays and Delta for a period representing 18 tidal cycles.

A comparison of model-prototype behavior indicated a slightly flatter salinity gradient tended to develop in the model than was observed in the prototype. Concentrations of salt tended to be slightly low in the seaward portion of the system and somewhat high in the landward portion. On the other hand, the distribution of total salt through the system was well represented by the model. There were at least four possible causes for this difference in salinity gradient, three of which were related to quality and quantity of prototype data. The fourth possibility was identified as "induced advective dispersion" associated with numerical procedures of integration. Relative contributions by each of these causes was difficult to assess and it was suggested that a sensitivity analysis of the models could prove valuable in determining the contributions of each.

In conclusion it can be stated that the Tidal Hydrodynamic Model and the Dynamic Water Quality Model are capable of simulating the prototype behavior of large two-dimensional estuarial systems for either conservative or non-conservative pollutants. Additional research and development in the areas of numerical methods of integration and sensitivity analysis should prove to be of great value in improving and assessing the capabilities of the existing models. The greatest limitation of these models is imposed by the lack of adequate data for verifying functionally the processes which affect water quality.

A great deal of time, money, and effort is presently being invested in an effort to improve the present level of understanding concerning the environment and its relation to water quality. It is expected that the hydraulic-water quality models developed in connection with FWPCA studies of the Bay-Delta System will be valuable tools for the enhancement of this understanding and for providing firm bases for decision making.